5.0 WAVE RUNUP, SETUP, AND OVERTOPPING

Wave runup is the uprush of water from wave action on a shore barrier intercepting stillwater level. The water wedge generally thins and slows during its excursion up the barrier, as residual forward momentum in wave motion near the shore is fully dissipated or reflected. The notable characteristic of this process for present purposes is the wave runup elevation, the vertical height above stillwater level ultimately attained by the extremity of uprushing water. Wave runup at a shore barrier can provide flood hazards above and beyond those from stillwater inundation and incident wave geometry, as sketched in Figure 11.

Two additional phenomena, wave setup and wave overtopping, may require explicit consideration for adequate treatment of the coastal flood hazards linked to wave runup. Wave setup generates a mean water surface elevated above the stillwater level, due to accumulation of water against a barrier exposed to wave heights attenuating in shallow water. Wave overtopping consists of any wave-induced flow passing over the barrier crest, so that flood water can provide wave-like impacts, sheet flow, and/or quiet ponding over an inland area. These phenomena and their quantitative evaluation will be addressed in later subsections, after describing the more basic assessment of wave runup for a coastal FIS.

The extent of runup can vary greatly from wave to wave in storm conditions, so that a wide distribution of wave runup elevations provides the precise description for a specific situation. Current policy for the NFIP

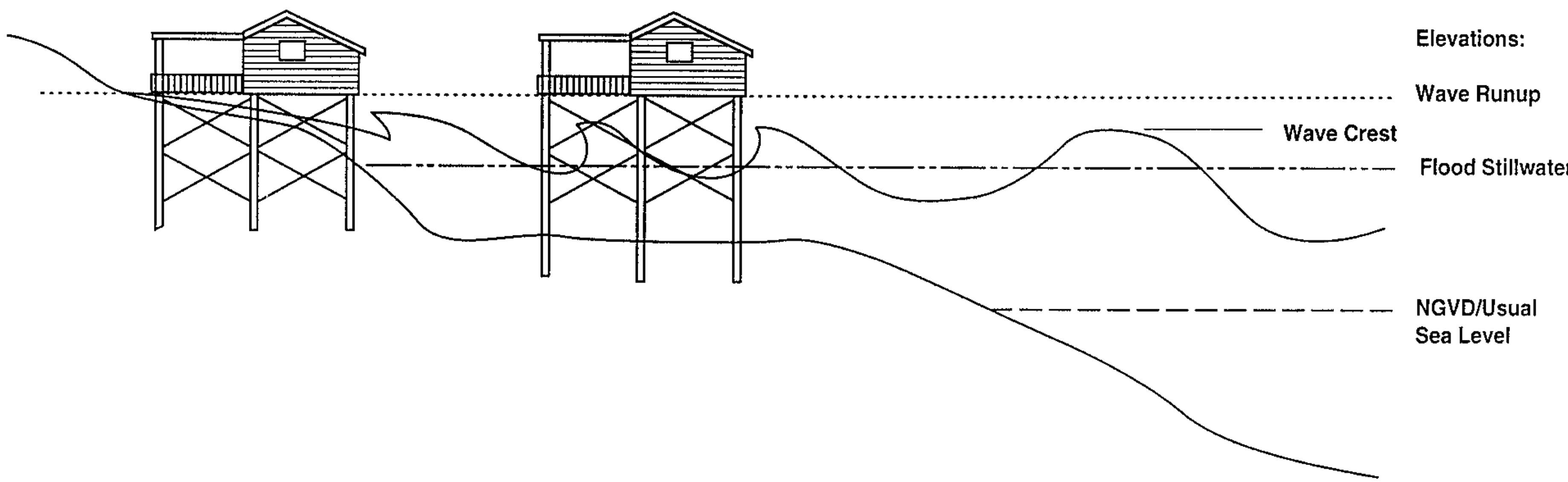

Figure 11. Schematic Illustration of Wave Effects Extending Above and landward of Stillwater Intercept on Transect.

is that the mean runup elevation (rather than some occasional extreme) for a situation is appropriate in mapping coastal hazards of the base flood. The following material describes content and usage of the Wave Runup Model, a FEMA computer program that determines mean runup elevation once the coastal flood situation is specified.

5.1 Wave Runup Model Description

The current version of the FEMA Wave Runup Model, called RUNUP 2.0, may be run either on a minicomputer (e.g., DEC VAX 11/750) or on an IBM-compatible personal computer (PC or PC/AT). Given the flood level, shore profile and roughness, and incident wave condition described in deep water, the program computes by iteration a wave runup elevation fully consistent with the most detailed guidance available (Reference 32). This determination includes an analysis separating the profile into an approach segment next to the steeper shore barrier, and interpolation between runup guidance for simple configurations bracketing the specified situation.

Some additional description of the workings of the Wave Runup Model can assist informed preparation of input and interpretation of output. The incorporated guidance gives runup elevation as a function of wave condition and barrier slope, for eight basic shore configurations distinguished by water depth at the barrier toe, along with the approach geometry. Where those basic geometries do not appropriately match the specified profile, reliance is placed on the composite slope method of Reference 33; this assumes the input shore profile (composite slope) is equivalent to a hypothetical uniform slope, as shown in Figure 12. The runup elevations are derived from laboratory measurements in uniform wave action, rather than the irregular storm waves usually accompanying a flood event. Runup guidance for uniform waves, however, also pertains to the mean runup elevation from irregular wave action with identical mean wave height and mean wave period. Figure 13 presents an overview of the basic computation procedure within RUNUP 2.0.

Basic empirical guidance incorporated within this computer model generally does not extend to vertical or nearly vertical flood barriers. For such configurations, RUNUP 2.0 usually will provide a runup elevation but the result may be misleading, because reliance on the composite-slope method can yield an underestimate of actual wave runup with the abrupt barrier. Where a vertical wall exists on a transect, it is preferable to develop a runup estimate using specific guidance in Figure 14, from the Shore Protection Manual (Reference 12). As within RUNUP 2.0, these empirical results for uniform waves should be utilized by specifying mean wave height and mean wave period for entry, and taking the indicated runup as a mean value in storm wave action. Shore configurations with a vertical wall are also addressed separately by detailed wave overtopping guidance presented in Section 5.7.

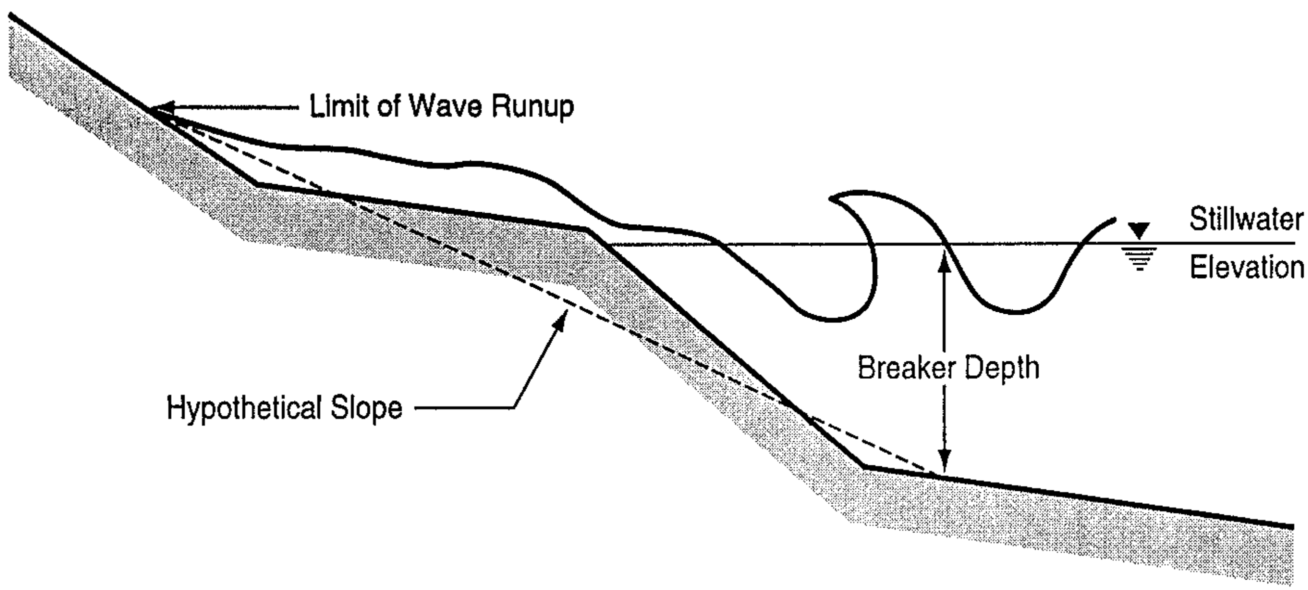

Figure 12. Hypothetical Slope for Determining Wave Runup on Composite Profiles.

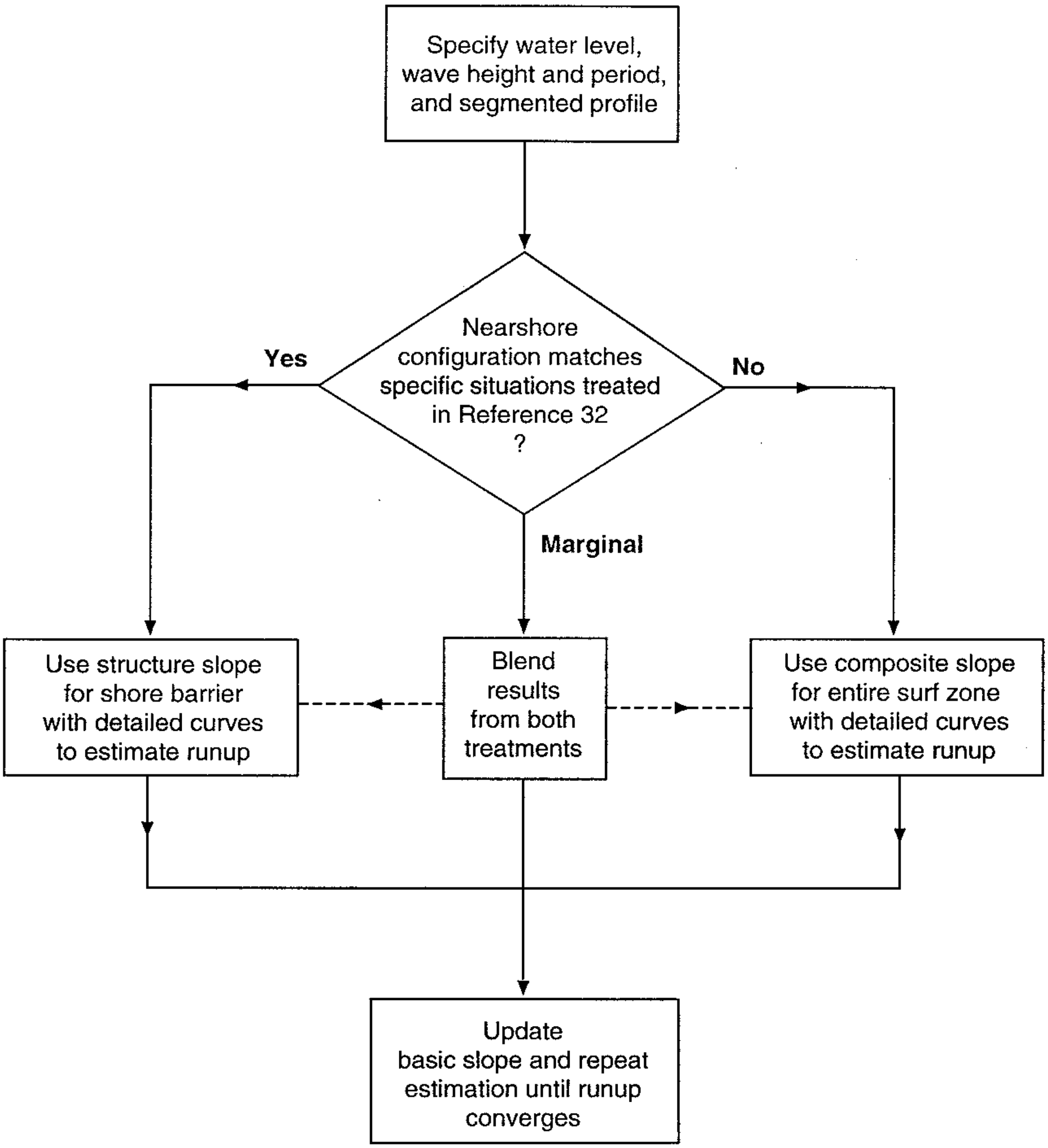

Figure 13. Overview of computation procedure implemented in modified FEMA Wave Runup Model.

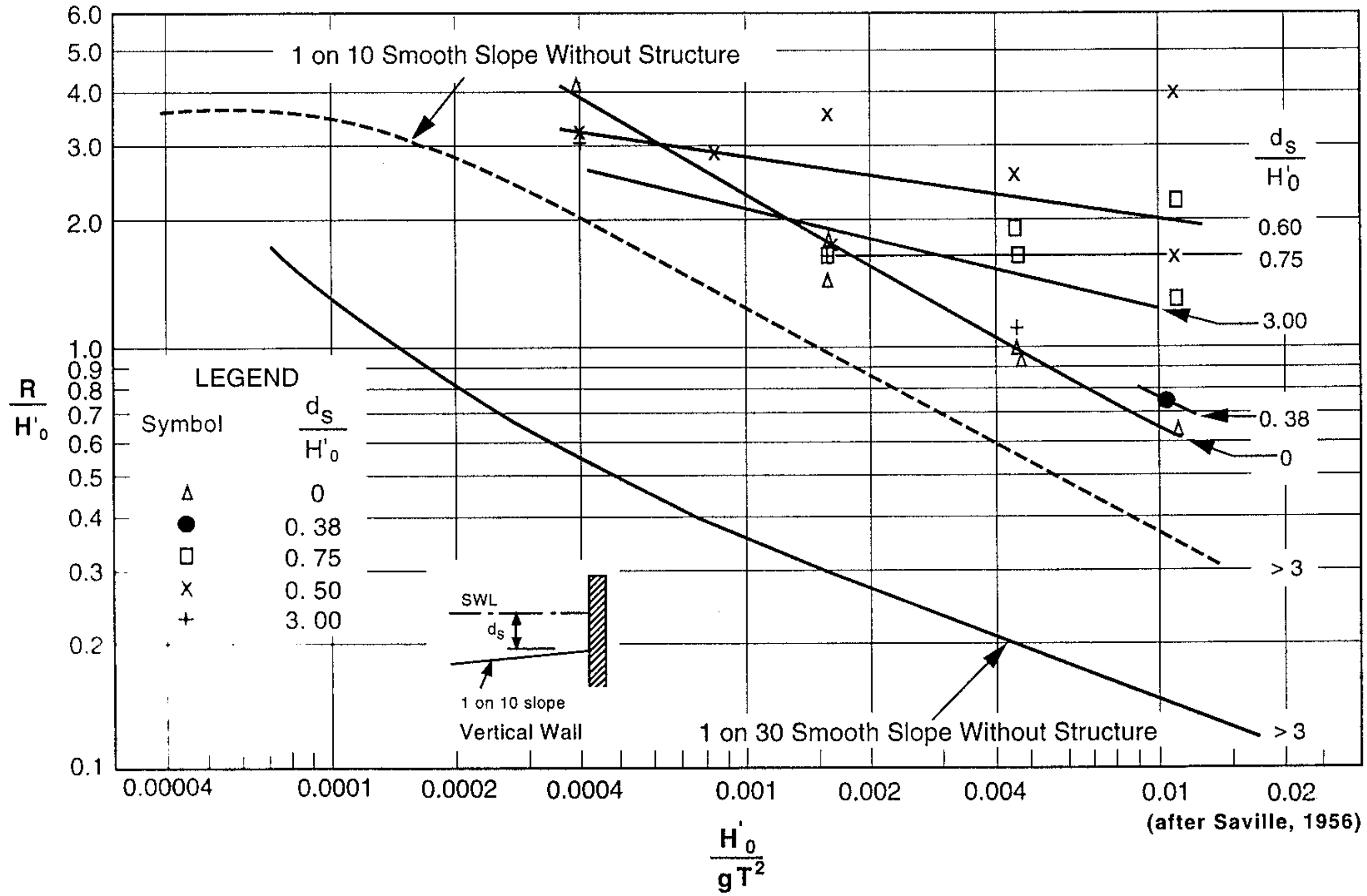

Figure 14. Wave Runup Guidance from Vertical Wall, From Reference 12.

5.2 Wave Runup Model Input Preparation

The input to the Wave Runup Model is done by transects. Transects should be located along the shoreline as previously specified. Because the runup results are very sensitive to shore slope or steepness, it is important to have at least one transect for each distinct type of shore geometry. Often, areas with similar shore slopes are located throughout a community, and the results of one transect can be applied to all the areas that are similar. This is especially typical of New England communities with rocky bluffs. When the Wave Runup Model is being applied to dune remnants where eroded slopes are fairly uniform, transect location should be governed by the upland land-cover characteristics which are major considerations in the WHAFIS model.

The ground profile for the transect is plotted from the topography and bathymetry after the data have been referenced to the National Geodetic Vertical Datum (NGVD). The profile should extend from an elevation below the breaker depth to an elevation above the limit of runup or to the maximum ground elevation. An adequate vertical extent for the transect description will usually be 1.5 times the wave height above and below stillwater elevation. If the landward profile does not extend above the computed runup (30 feet NGVD is commonly a maximum), it will be assumed that the last positive slope segment continues indefinitely. This is very common with low barriers, so the last slope should be carefully chosen to be representative. To complete the description, each slope segment of the profile will need a roughness coefficient, with some common values presented in Table 3. Roughness coefficient must be between zero (maximum roughness) and one (hydraulically smooth), and values for slope segments above stillwater level control the estimated runup. The roughness coefficient *(r)* is used as a multiplier for runup magnitude *(R)* defined on a smooth barrier to estimate wave runup with a rough barrier.

Transects are approximated by the minimum adequate number of linear segments, up to 20 as a limit. Segments may be horizontal, or higher at the landward end; portions with opposite inclination should be represented as horizontal when developing the transect approximation. Using many linear segments to represent a transect can be wasted effort, since the Wave Runup Model may combine adjacent segments in defining the appropriate approach and barrier extents. Bearing in mind the runup computation procedure, engineering judgment applied to transect representation can assist in obtaining the most valid estimate of wave runup elevation.

The input transect should reflect wave-induced modifications expected during the 100-year event, including erosion on sandy shores with dunes. Only coastal structures expected to remain intact throughout the 100-year event should be represented on a specific transect. Besides the transect specification, other required input data for the Wave Runup Model are the 100-year

Table 3. Values for Roughness Coefficient in Wave Runup Computations

ROUGHNESS COEFFICIENT	DESCRIPTION OF BARRIER SURFACE
1.00	Sand; smooth rock, concrete, asphalt, wood, fiberglass
0.95	Tightly set paving blocks with little relief
0.90	Turf, closely set stones, slabs, blocks
0.85	Paving blocks with sizable permeability or relief
0.80	Steps; one stone layer over impermeable base; stones set in cement
0.70	Coarse gravel; gabions filled with stone
0.65	Rounded stones, or stones over impermeable base
0.50	Cast-concrete armor units: cubes, dolos, quadripods, tetrapods, tribars, etc.

stillwater elevation and the incident mean wave condition described in deep water. The specified stillwater elevation should exclude any contributions from wind-wave effects. If available elevations include wave setup, that component should be removed prior to using this model so that calculated runup elevations do not indicate a doubled wave setup. Basic empirical guidance refers runup at a barrier to the water level in the absence of wave action, and thus includes the wave setup component.

The mean wave condition to be specified for valid results with the Wave Runup Model may be derived from other common wave descriptions by simple relationships. Wave heights in deep water generally conform to a Rayleigh probability distribution, so that mean wave height equals 0.626 times either the significant height based on the highest one-third of waves, or the zero-moment height derived from the wave energy spectrum. There is no exact correspondence between period measures, but mean wave period usually can be approximated as 0.85 times the significant wave period or the period of peak energy in the wave spectrum.

Table 4 lists a series of wave height and period combinations, of which one should be fairly suitable for runup computations at fully exposed coastal sites (depending on the local storm climate). These mean wave conditions have wave steepness values typical of U.S. hurricanes, or within 30% of a fully arisen sea for extratropical storms. Commonly, there may be some difficulty in specifying a

Table 4. Appropriate Wave Conditions for Runup Computations Pertaining to 100-Year Event in Coastal Flood Insurance Studies

Mean Wave Period (sec)	Mean Deep-Water Wave Height (ft)
Hurricanes	
8	12
9	15½
10	19
11	23
12	27½
Extratropical Storms	
11	18
12	21½
13	25
14	29
15	33½

precise wave condition as accompanying the 100-year flood. In that case, it is appropriate to consider also wave heights and periods both 5% higher and lower than that selected (or whatever percentages suit the level of uncertainty), and to run the model with all nine combinations of those values. The average of computed runup values then provides a suitable estimate for mean runup elevation. A wide range in computed runups signals the need for more detailed analysis of expected wave conditions or for reconsideration of the transect representation.

5.3 Wave Runup Model Operation

The input to the FEMA Wave Runup Model consists of several separate lines specifying an individual transect and the hydrodynamic conditions of interest within particular columns. All input information is echoed in an output file, which also includes computed results on wave breaking and wave runup.

The input format is outlined in Table 5. The first two lines of the input give the Name and Job Description, which must be included for each transect. The next line of input is the Last Slope, which contains the cotangent of the shore profile continuing from the most landward point provided. This is followed by the profile points which define the nearshore profile in consecutive order from the most seaward point. Each line gives the elevation and station of a profile point and the roughness coefficient for the segment between

Table 5. Description of the five types of input lines for Wave Runup Model

Name Line

This line is required and must be the first input line.

Columns	Contents
1-2	Blank
3-28	Client's Name
29-60	Blank
61-70	Engineer's Name
71-80	Job Number

Job Description Line

Columns	Contents
1-2	Blank
3-76	Project description or run identification
77-80	Run Number

Last Slope Line

This line is required and defines the slope immediately landward of the profile actually specified in detail.

Columns	Contents
1-4	Slope (horizontal over vertical or cotangent) of profile continuation
5-80	Blank

Profile Lines

These lines must appear in consecutive order from the most seaward point landward. Each line has the elevation and station of a profile point and the roughness coefficient for the section between that point and the following point. The roughness coefficient on the last profile line is for the continuation defined in the Last Slope Line. At least one profile point with a ground elevation greater than the stillwater elevation must be specified. The number of Profile Lines cannot exceed 20.

Table 5 (continued)

Columns	Contents
1	Last point flag. The most landward point on the profile is indicated by a 1. If not the last point, leave blank.
2	Blank
3-7	Elevation with respect to NGVD, in feet
8	Blank
9-14	Horizontal distance. It is common to assign the shoreline (elevation 0.0) as Point 0 with seaward distances being negative and landward distances positive.
15	Blank
16-20	Roughness coefficient in decimal form between 0.00 (most rough) and 1.00 (smooth).
21-80	Blank

Water Level and Wave Parameter Lines

These lines specify hydrodynamic conditions for runup calculations on each profile. Namely, 100-year stillwater elevation along with mean wave height and period for deep water. Typically, stillwater elevation remains constant for a given profile, while the selected wave conditions closely bracket that expected to accompany the 100-year flood. A maximum of 50 of these lines can be input for each profile.

Columns	Contents
1	Last line, new transect flag. A 1 indicates the last line for a given transect and notifies that another transect is following. If not the last line, or if the last line of the last transect, leave blank.
2-6	Stillwater elevation with respect to NGVD, in feet.
7	Blank
8-12	Deepwater mean wave height, H_0, in feet, greater than 1 foot
13	Blank
14-18	Mean wave period, T, in seconds
19-80	Blank

that point and the following point. The roughness coefficient on the last profile line is for the continuation defined in the Last Slope line. The number of profile points cannot exceed 20. The final input is the series of hydrodynamic conditions of interest. Each line here contains the stillwater elevation along with a mean wave height in deep water and a mean wave period.

The output as shown in Table 6 has two parts. The first page is a printout of the transect listed as a numbered set of profile points, cotangents (slopes) of the segments, and the roughness coefficient for each segment. The second page is the output table of computed results for each set of conditions: the values of runup elevation and breaker depth, each with respect to the specified stillwater elevation, along with an identification of the segment numbers giving the seaward limit to wave breaking and the landward limit to mean wave runup.

5.4 Wave Runup Model Output Messages

There are several output messages that alert the user to specific problems encountered in running the program. All but the last three indicate that the program has stopped execution without completing runup calculations.

```
CLIENT- FEMA                     ** WAVE RUNUP-VERSION 2.0  ** ENGINEERED BY DAB       JOB 1
PROJECT-Runup calculations on Eroded Transect                                       RUN 1    PAGE  1

************************************************************************************************************************


                    CROSS SECTION PROFILE

                  LENGTH   ELEV.     SLOPE      ROUGHNESS

     1           -2670.0   -34.0
                                      97.50         1.00
     2           -1500.0   -22.0
                                      76.25         1.00
     3            -585.0   -10.0
                                      72.50         1.00
     4            -150.0    -4.0
                                      36.43         1.00
     5             -99.0    -2.6
                                      42.22         1.00
     6              53.0     1.0
                                      24.64         1.00
     7             223.0     7.9
                                      39.60         1.00
     8             322.0    10.4
                                        .99         1.00
     9             335.0    23.5
                                      10.00         1.00
    10             350.0    25.0

                       LAST SLOPE   1.00     LAST ROUGHNESS   1.00
```

Table 6. Output Example for the FEMA Wave Runup Model

****

OUTPUT TABLE

INPUT PARAMETERS			RUNUP RESULTS			
WATER LEVEL ABOVE DATUM (FT.)	DEEP WATER WAVE HEIGHT (FT.)	WAVE PERIOD (SEC.)	BREAKING SLOPE NUMBER	RUNUP SLOPE NUMBER	RUNUP ABOVE WATER LEVEL (FT.)	BREAKER DEPTH (FT.)
10.40	16.60	10.30	2	8	1.66	26.46
10.40	16.60	10.80	2	8	1.83	26.83
10.40	16.60	11.30	2	8	1.99	27.20
10.40	17.50	10.30	2	8	1.75	27.69
10.40	17.50	10.80	2	8	1.92	28.07
10.40	17.50	11.30	2	8	1.92	28.45
10.40	18.40	10.30	2	8	1.84	28.91
10.40	18.40	10.80	2	8	1.84	29.30
10.40	18.40	11.30	2	8	2.02	29.69
					Avg. 1.89	

- "NEGATIVE RUN PARAMETER, PROGRAM STOPS"

  An input value of wave height or wave period is read as negative or zero. Check that the input has been entered in the correct columns.

- "MORE THAN 20 POINTS IN PROFILE, PROGRAM STOPS"

  The program accepts a maximum input of 20 points defining the nearshore profile. This encourages a profile approximation that is not overly detailed, since each transect is to represent an extensive area.

- "**** H_o/L_o LESS THAN 0.002 ****"
- "**** H_o/L_o GREATER THAN 0.07 ****"

  These limits on wave steepness pertain to the extent of incorporated guidance on breaker location. They should be adequate to include appropriate mean wave conditions for extreme events, and also conform to the usual limits in detailed guidance on wave runup elevations.

- "DATA EXCEEDED TABLE"

  An entry into subroutine LOOK of the program is not within the parameter bounds of the data table from which a value is sought.

- "SOLUTION DOES NOT CONVERGE"

  After ten iterations, the current and previous estimates of runup elevation continue to differ by more than 0.15 foot, and both values are provided in the output table. The calculation is usually oscillating between these two runup estimates when this occurs.

- "COMPOSITE SLOPE USED BUT WAVE MAY REFLECT, NOT BREAK"

  The output runup elevation relies to some extent on a composite-slope treatment, but the overall slope is steep enough that the specified wave may reflect from the nearshore barrier. Thus, the application of a calculated breaker depth in determining overall slope and runup elevation is questionable.

- "WARNING; COMPOSITE SLOPE USED, BUT INPUT PROFILE DOES NOT EXTEND TO BREAKER DEPTH"

  If the input profile does not extend seaward of the breaker depth, an incorrect breaker depth may be computed and the associated runup elevation will also be incorrect. The input profile should include bathymetry to 30 or 40 feet in depth.

5.5 Wave Runup in Special Situations

Output of the Wave Runup Model should be examined carefully for each distinct situation, to assist proper interpretation and application of calculated results. One important consideration is that a mean runup elevation below the crest of a given barrier does not necessarily imply the barrier will not occasionally be overtopped by flood waters; the necessary supplementary examination of wave overtopping is addressed in Section 5.7 below. Other cases may yield results of more immediate concern, in that the Wave Runup Model may calculate a runup elevation exceeding maximum barrier elevation; this outcome can occur because the program assumes the last positive slope to continue indefinitely. The following material provides guidance on proper assessment of flood hazards beyond relatively low shore barriers, where wave runup surpasses the maximum ground elevation but falls off before it reaches the computed runup elevation.

For bluffs or eroded dunes with negative landward slopes, a general rule has been used that limits the wave runup elevation to 3 feet above the maximum ground elevation. When the runup overtops a barrier such as a partially eroded bluff or a structure, the flood water percolates into the bed and/or runs along the back slope until it reaches another flooding source or a ponding area. The runoff areas are usually designated as Zones AO with a depth of flooding given (1, 2, or 3 feet). Ponding areas are designated as Zone AH (depth of flooding equal to 3 feet or less) with a flood elevation given. Standardized NFIP procedures have been developed for the treatment of sizable runoff and ponding, but are beyond the scope of this presentation; see Reference 1.

A fairly typical situation on Atlantic and Gulf coasts is that wave runup exceeds the barrier top and flows to another flooding source such as a bay, river, or backwater. It may not be necessary in this situation to compute overtopping rates and ponding elevations; only the flood hazard from the runoff needs to be determined. Simplified procedures have been used to determine an approximate depth of flooding in the runoff area (Reference 34). These procedures are illustrated on Figure 15 and discussed below.

When the runup computed on the imaginary extension of the last positive slope is equal to or greater than 3 feet above the maximum ground elevation, the maximum runup is taken to be 3 feet above the ground crest elevation. This elevation decays to 2 feet above the ground profile at 50 feet behind the crest, and continues at this depth until it encounters other flooding. Computed runup is not adjusted if it is less than 3 feet above the ground crest. In the same initial 50 feet, this elevation decays to one foot above the ground and continues at this depth until it encounters other flooding. The runoff area from the ground crest to the limit of the other flooding is designated Zone AO with the appropriate depth of flooding specified.

A distinct type of overflow situation can arise at low bluffs or banks backed by a nearly level plateau, where calculated wave runup may appreciably exceed the top elevation of the steep barrier. Reference 35 provides a simple procedure to determine realistic runup elevations for such situations, as illustrated in Figure 16. An extension to the bluff face slope permits computation of a hypothetical runup elevation for the barrier, with the imaginary

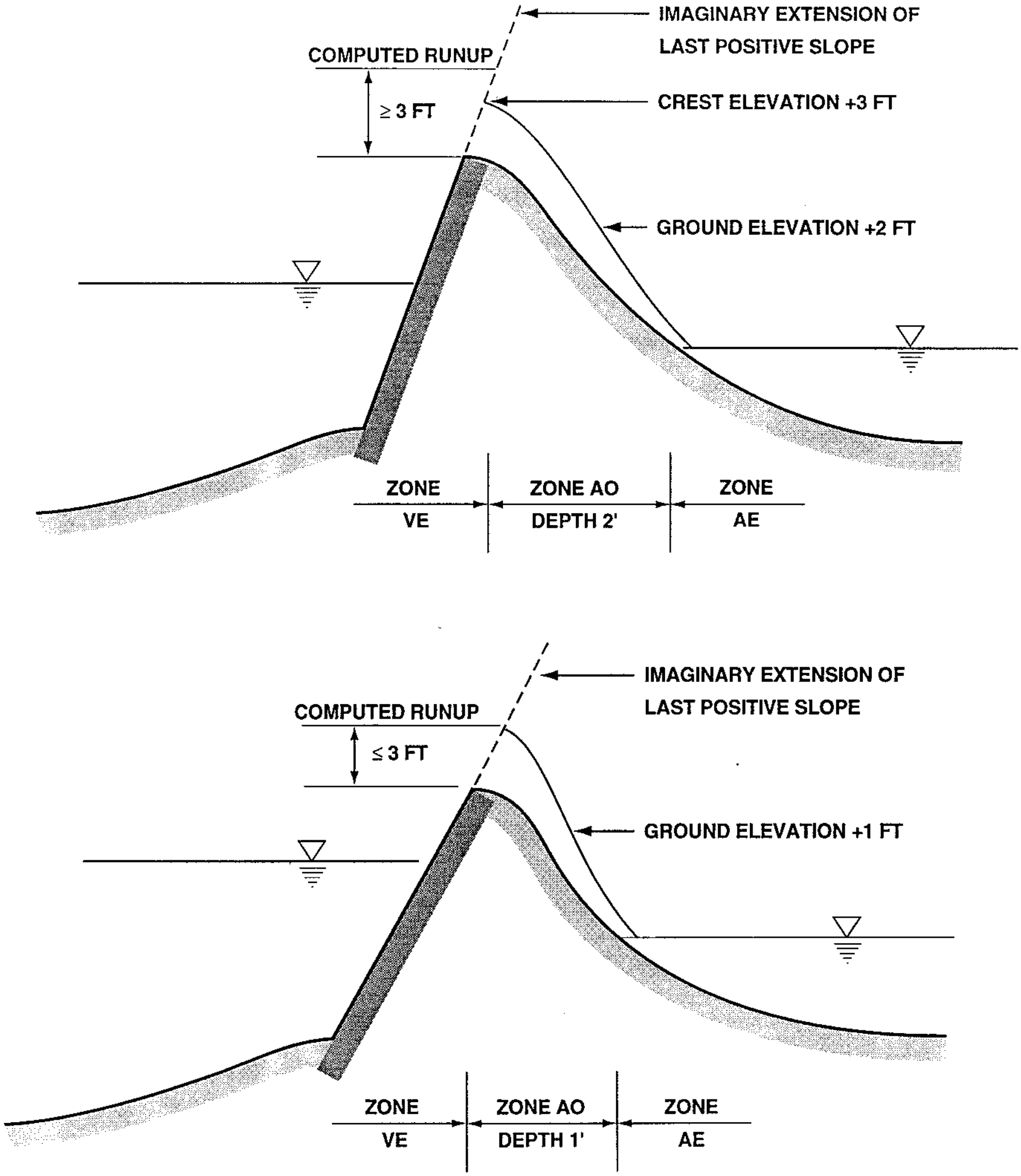

Figure 15. Simplified Run-off Procedures (Zone AO).

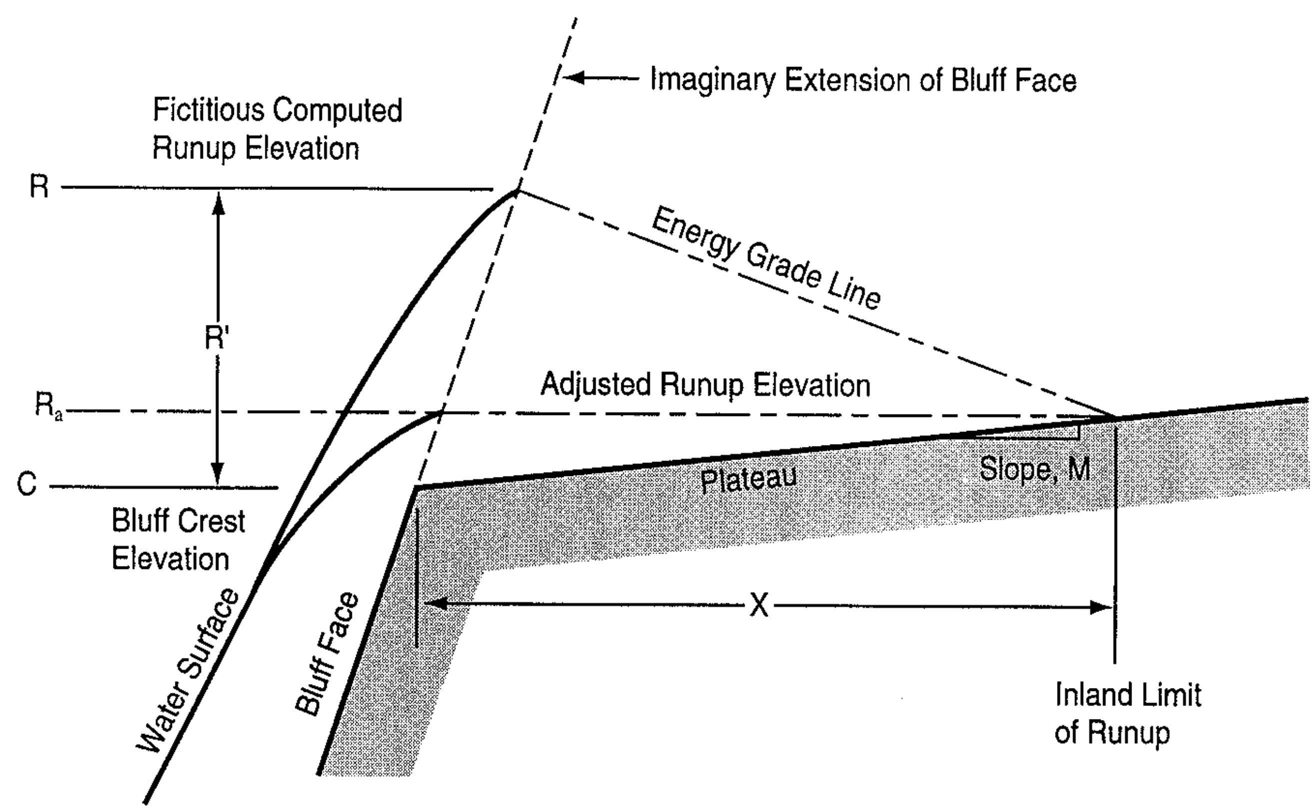

Figure 16. Treatment of Runup onto Plateau above Low Bluff.

portion given by the excess height $R' = (R\text{-}C)$ between calculated runup and the bluff crest. Using that height R' and the plateau slope m, Figure 17 defines the inland limit to wave runup, X, corresponding to runup above the bluff crest of $(m\,X)$ or an adjusted runup elevation of $R_a = (C + mX)$. This procedure is based on a Manning's "n" of 0.04 along with some simplifications in the energy grade line, and is meant for application only with positive slopes landward of the bluff crest. Reference 36 provides a different treatment of wave overflow onto a level plateau, for possible FIS usage.

These runup assessment procedures are given for general guidance, but situations may exist where they are not entirely applicable. For example, runup elevations need to be fully consistent with wave setup and wave overtopping assessments described in the following sections. In problematic cases, good judgment and reliance on the historical data should be used to reach a solution about realistic flood hazards associated with a shore barrier. Chapter 7 considers the integration of separately calculated wave effects into coherent hazard zonations for the base flood. When a unique situation is encountered, a Special Problem Report should be prepared and discussed with the Project Officer.

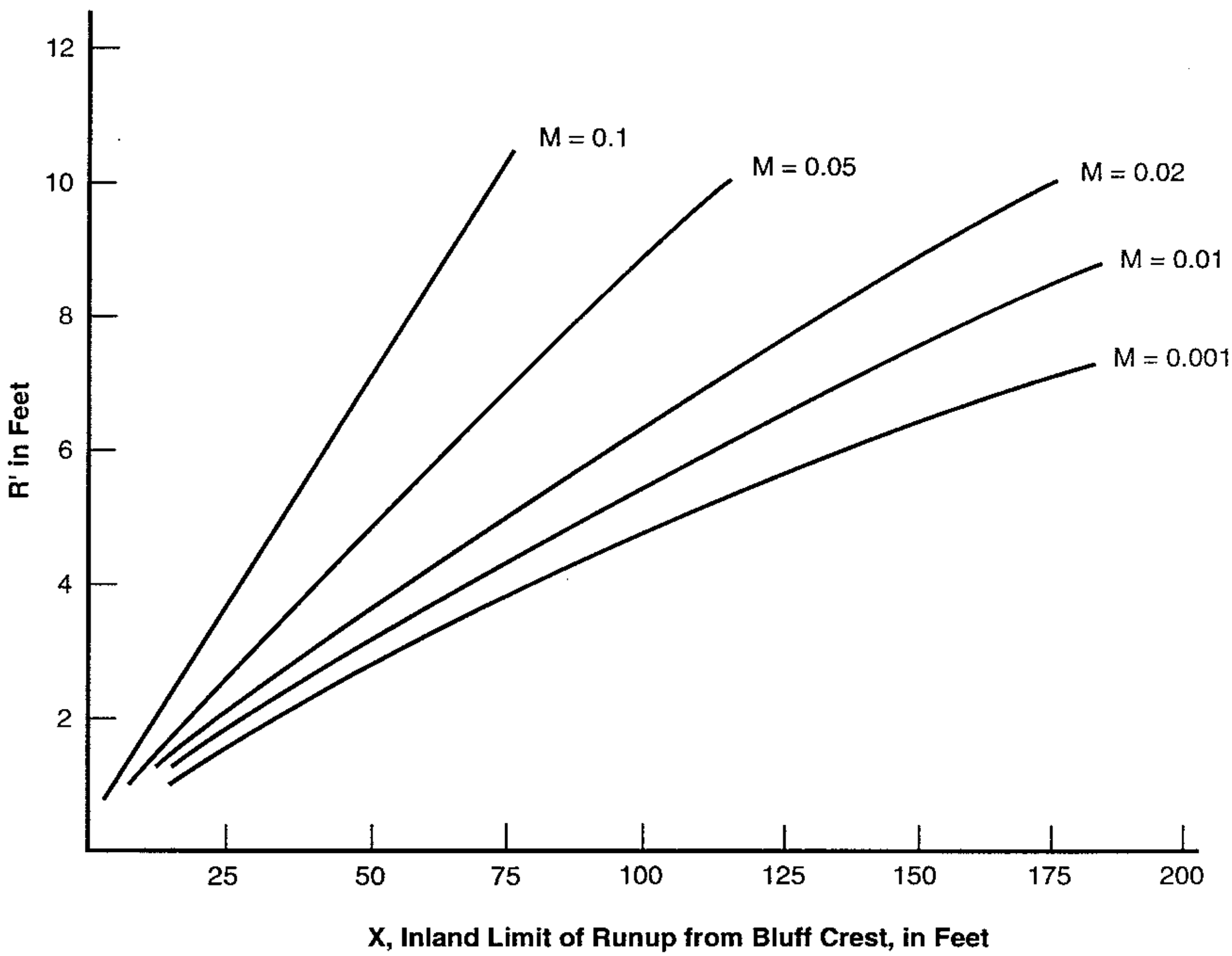

Figure 17. Curves for Computation of Runup Inland of Low Bluffs.

5.6 Wave Setup

Nearshore wave action can increase mean water elevation in front of a shore barrier by the phenomenon called wave setup, which is related to wave attenuation by breaking in shallow water. In treating the 100-year flood, focus may be restricted to the cumulative setup effect in the immediate vicinity of the shore barrier. Laboratory measurements of wave runup generally include the contribution due to wave setup, because runup elevations are defined relative to stillwater level in the absence of wave action.

A separate calculation for wave setup can be appropriate even if a wave runup elevation has already been determined, in part because the changed mean water depth can increase wave heights and crest elevations to be expected near the shore. In addition, empirical guidance within the Wave Runup Model is based on uniform laboratory wave action, so that incorporated setup might pertain to the field situation of swell waves from distant storms; setup effects may be much different in the local storm waves accompanying the 100-year coastal flood. If storm wave setup is found to exceed the wave runup calculated for a particular situation, the setup estimate must be applied as a lower bound for actual wave runup in further analysis of wave effects and base flood elevations.

Reference 12 provides straightforward empirical guidance on wave setup for various storm wave conditions and plane bottom slopes, as reproduced in Figure 18. Setup magnitude here is given in dimensionless form, as normalized by incident significant wave height. This guidance with typical significant storm-wave steepnesses about 0.03 to 0.04 indicates shore setups amounting to 7% or 8% of incident wave height. Incident wave conditions are specified in deep water as the significant wave height and the wave steepness, $H_{os}/L_{op})$, where $L_{op} = gT_p^2/2\pi$ is wavelength in deep water. Bottom slope may be taken as an overall average over the breaker zone between $d = 2H_o$ and d=0, if the bottom geometry is relatively simple. For other geometries, e.g., with a berm or reef in front of the shore barrier, the wave setup can be larger than given by Figure 18 and a more detailed examination may be required.

Wave setup also appears appreciably larger according to an independent treatment of storm waves on plane slopes, as outlined in Reference 26 for a relatively narrow spectrum describing incident wave energy. If historical evidence indicates greater setup increases of mean water depth in extreme floods than Figure 18 gives for the study site, a wave setup estimate based on that independent guidance may be conveniently developed through an ACES computer program provided in Reference 27. The program does not permit direct calculation of wave effects at d=0, but setup results from about $d=H_o$ to the shallow limit of computations may be linearly extrapolated to the stillwater shoreline.

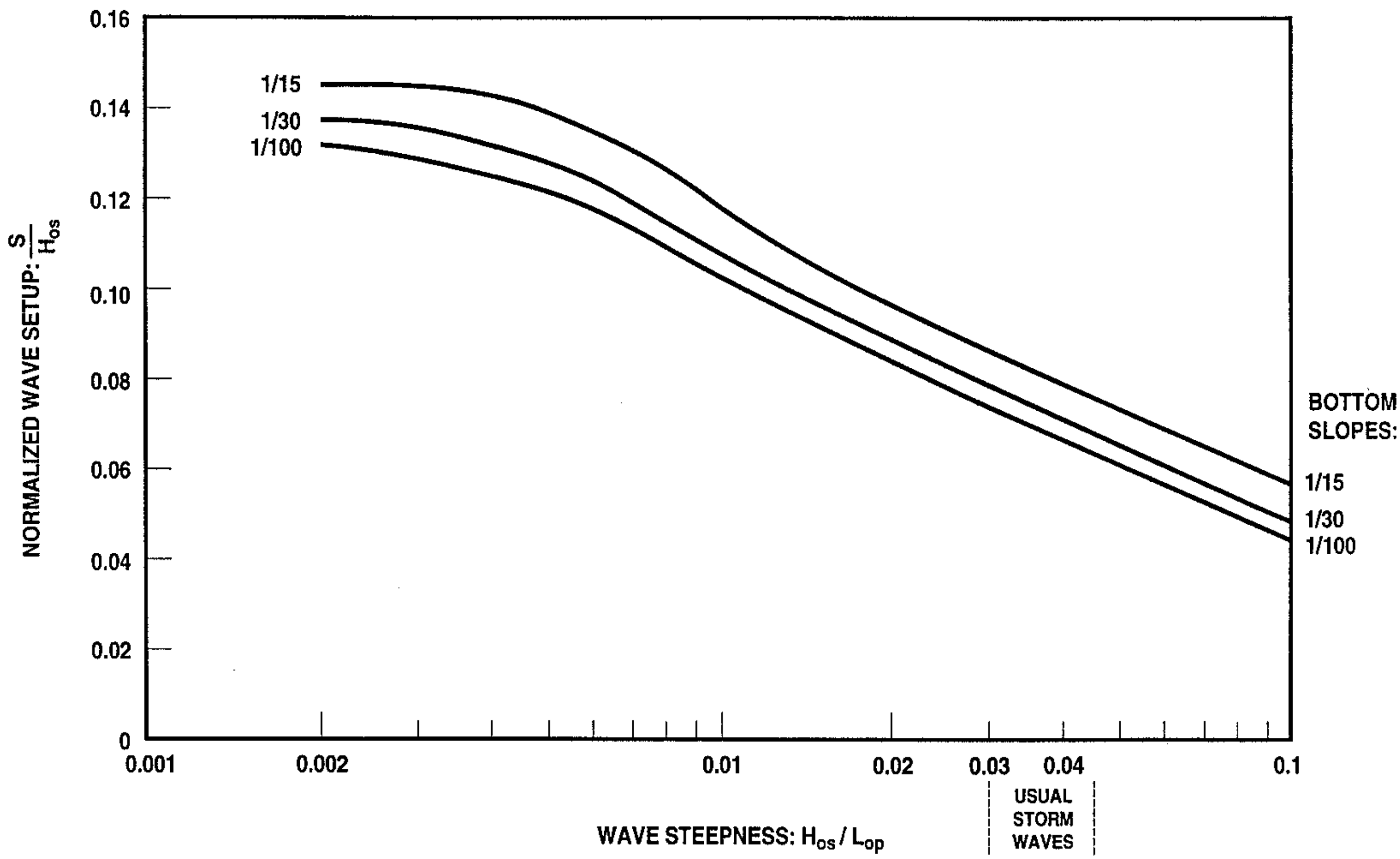

Figure 18. Guidance on Wave Setup in Irregular Wave Action, From Shore Protection Manual (Reference 12).

5.7 Wave Overtopping

Wave overtopping results when a shore barrier does not contain incident wave action, so that flood water penetrates to the protected area landward. This process of a partial halt and dissipation to storm waves is more difficult to treat than wave runup or wave setup. Important rates of wave overtopping can vary over several orders of magnitude, and can depend strongly on the detailed geometry of the barrier. That complicates the development of empirical guidance on wave overtopping, but there apparently is little demand for such guidance in coastal engineering practice. According to Reference 17, the design process for any major coastal flood-protection structure relies on site-specific model testing, rather than generalized overtopping guidance.

Of course, the assessment of potential wave overtopping for present purposes must rely on readily available empirical guidance, historical effects, and engineering judgment. Except for very heavy overtopping, useful guidance must be derived from tests with irregular waves, because the intermittently large overtopping discharges in storm situations could not be reproduced otherwise. Adding to the formal complexity of an adequate treatment for flood hazard assessment, overtopping effects may be cumulative so that the entire course of a flood event could require consideration, not just the peak conditions. Fortunately, only the order of magnitude of overtopping rates commonly needs to be estimated because there are clearly documented thresholds below which wave overtopping may be classified as negligible. On the other hand, it must be noted that if a preliminary estimate indicates severe overtopping which threatens the stability of a given structure, then that structure might be removed from the transect for analyses of the base flood, so no further overtopping consideration is required.

References 26 and 31 appear to provide the most trustworthy and wide-ranging summaries of mean overtopping rates with storm waves. Reference 31 addresses smooth plane or bermed slopes, and Reference 26 considers vertical walls with or without a fronting rubble mound. Before surveying those primary sources of overtopping guidance, however, some introductory considerations can help to determine whether detailed assessment is needed for base flood conditions at a specific shore barrier.

The initial consideration should be an interpretation of mean runup elevation already calculated ($\bar{R}$), in terms of likely extreme elevations according to the Rayleigh probability distribution usually appropriate for wave runups. To parallel the extreme wave height addressed in coastal studies (Reference 5), a controlling runup magnitude may be defined as 1.6 times significant runup, or 2.5 times mean runup according to the Rayleigh distribution. If elevation of the barrier crest above 100-year stillwater elevation, or the barrier freeboard F, equals or exceeds (2.5 $\bar{R}$), then the landward area is not subject to wave-induced discharges in the base flood. That requirement might be supplemented by consideration of F near $(2\ \bar{R})$, corresponding to 4.5% of runups reaching the barrier crest according to the Rayleigh distribution. If $F \leq (2\bar{R})$, wave overtopping can certainly be appreciable during the base flood, and ponding or runoff behind the barrier should be assessed. Note that extreme runups introduced here, $(2\bar{R})$ and $(2.5\bar{R})$, bracket the elevation exceeded by the extreme 2% of wave runups, a value commonly considered in structure design.

Once the need for quantitative overtopping assessment is established, wave runup considerations become inapplicable because a runup elevation generally cannot be converted to an overtopping estimate. Also, the composite-slope method used in determining wave runup does not appear applicable for overtopping of barriers with composite geometry, because details of the wave transformation on a barrier influence the resultant overtopping rates. Wave overtopping estimates for a specified situation generally must be based on measurements in a similar configuration. Before considering some implications of quantitative guidance for idealized cases, an overview of overtopping magnitudes gives a useful introduction (References 26, 37).

Wave overtopping is specified as a mean discharge: water volume per unit time and per unit alongshore length of the barrier, commonly cfs/ft. Interpreting or visualizing a given overtopping rate should take into account that the actual discharges generally are intermittent and isolated, being confined to some portion of occasional wave crests at scattered locations. Distinct regimes of wave overtopping may be described as spray, splash, runup wedge, and waveform transmission, in order of increasing intensity. Water discharges corresponding to those regimes naturally depend on the incident wave size, but certain overtopping rates have been identified with various impacts (Reference 26). Among those rates, 0.01 cfs/ft seems to correspond to flooding that generally should be considered appreciable, and 1 cfs/ft appears to define an approximate threshold where structural stability of the shore barrier commonly becomes threatened by severe overtopping.

Once mean overtopping rate has been estimated for the base flood, determining resultant flooding may require a representative duration for the interval of overtopping. That duration can vary widely depending on the coastal flood cause, from a fast-moving hurricane to a nearly stationary extratropical storm (Figure 2). A minimum assumption for the duration of flood-peak overtopping would generally be one to two hours. Durations on the order of ten hours or more could be appropriate for cumulative effects in an extratropical storm causing flooding over multiple high tides.

Figure 19 summarizes some empirical overtopping guidance for storm waves, in a schematic form meant to assist deciding the likely significance of flooding behind a coastal structure. Variables describing the basic situation are cotangent of the front slope for

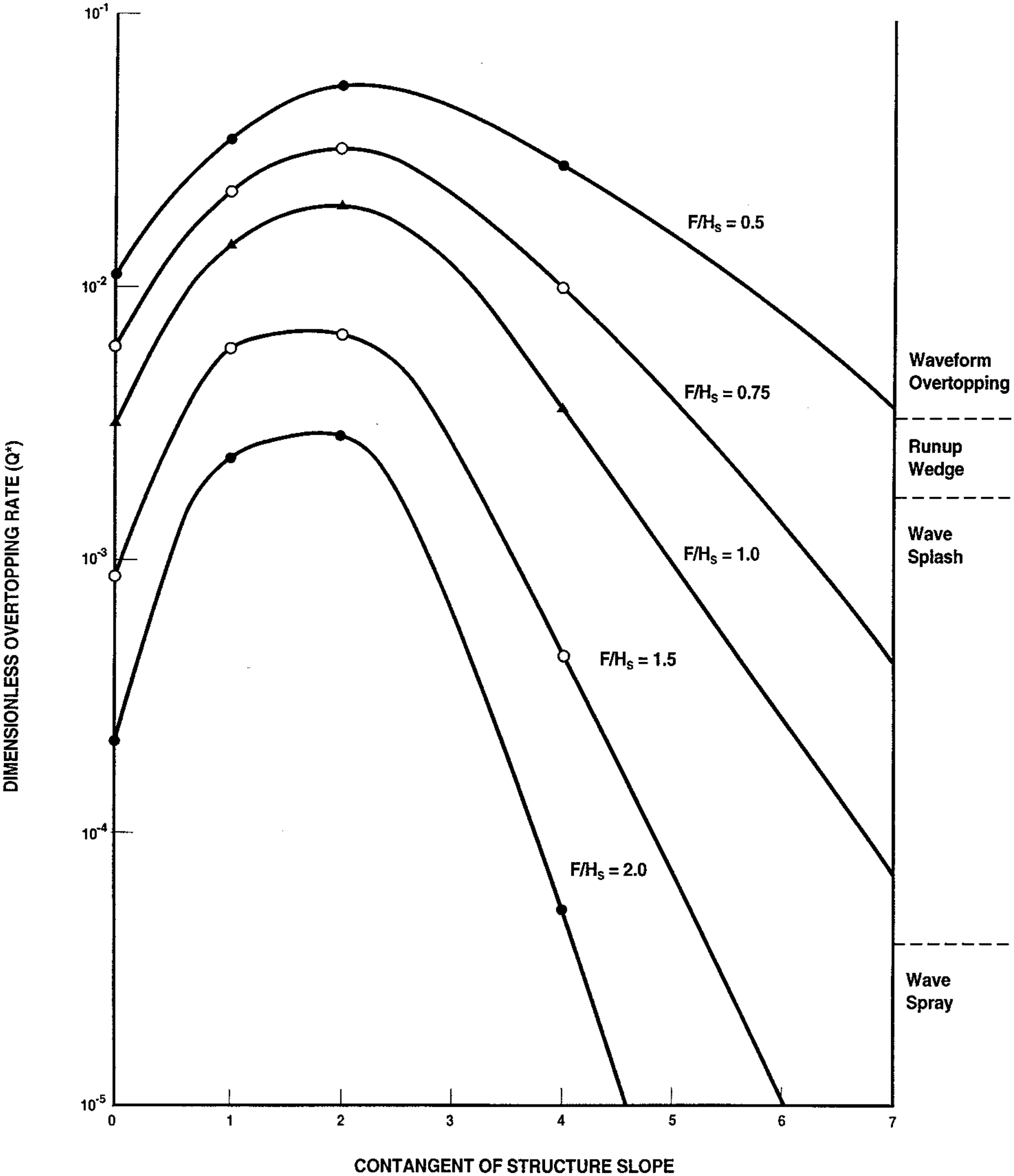

Figure 19. Schematic Summary of Storm-Wave Overtopping at Structures of Various Slopes and Freeboards, Based on References 26 and 31.

a smooth structure with ideally simple geometry, and freeboard of the structure crest above stillwater level, as normalized by incident significant wave height, F/H_s. The mean overtopping rate, $\overline{Q}$, is provided in dimensionless form as

$$Q^* = \overline{Q}/(gH_s^3)^{0.5}, \tag{2}$$

with test results shown for structure slopes of 1 on 1, 1 on 2, and 1 on 4 (Reference 31), and for a smooth vertical wall (Reference 26). These results pertain to: significant wave steepness of about $2\pi H_s/gT_p^2 = 0.035$, fairly appropriate for extreme extratropical storms or hurricanes; water depth near the structure toe of about $d_t = 2H_s$, so that incident waves are not appreciably attenuated; and moderate approach slopes, of 1 on 30 for a vertical wall, or 1 on 20 for other structures. The major feature of interpolated curves is fixed as a maximum in overtopping rate for structure slope of 1 on 2, corresponding to the gentlest incline producing (at this wave steepness) total reflection rather than breaking, and thus peak waveform elevations (Reference 38).

These measured results for smooth and simple geometries clearly show severe or "green water" overtopping even at relatively high structures ($F \geq H_s$) for a wide range of common inclinations (cotangents between about 0 and 4). Also, for freeboards considered here, a vertical wall (cotangent 0) permits less overtopping than common sloping structures with cotangent less than about 3.5.

Gentler barriers are uncommon because the construction volume increases with the cotangent squared, so steep coastal flood-protection structures usually face attenuated storm waves and/or have rough surfaces. Basic effects of those differences can be outlined for use in simplified overtopping assessments.

For sloping structures sited within the surf zone ($d_t < 2H_s$), Reference 31 indicates that basic overtopping guidance in Figure 19 can be used with attenuated rather than incoming wave height. A simple estimate basically consistent with other analyses of the base flood is that significant wave height is limited to $H_s' = d_t/2$ at the structure toe. The value of $(2F/d_t)$ describes the effectively increased freeboard in entering Figure 19, and the indicated Q* value is then converted to $\bar{Q}$ using H_s'. Note that the presumed wave attenuation ignores any wave setup as a small effect with the partial barrier, and that d_t should always correspond to the scour condition expected in wave action accompanying the base flood.

Figure 19 might also be made applicable to rough slopes, using a roughness coefficient (r) from Table 3 to describe the effectively increased freeboard with greater wave dissipation on the structure. Reference 31 proposed that effect of structure roughness be formulated as F/r, and Reference 29 confirmed a similar dependence of overtopping on roughness in measured results for irregular waves. The overtopping relation reported as reliable in Reference 39 is

$$Q* = 8 \cdot 10^{-5} \exp[3.1(rR* - F/H_s)] \quad (3)$$

where $R* = [1.5\ m/(H_s/L_{op})^{0.5}]$, up to a maximum value of 3.0, is an estimated extreme runup normalized by H_s, for a barrier slope given as the tangent m. Equation 3 is meant to pertain to very wide ranges of test situations with moderate overtopping, but appears very approximate in comparison with specific results for r=1 shown in Figure 19. It may be advisable to evaluate Equation 3 for both smooth and rough barriers, then use the ratio to adapt a Figure 19 value for the case with roughness. Note that References 31 and 39 provide further overtopping guidance on the effects of composite profiles, oblique waves, and shallow water with sloping structures.

For overtopping of vertical walls, effects of wave attenuation appear relatively complex, but Reference 26 provides extensive empirical guidance on various structure situations with incident waves specified for deep water. Figure 20 converts basic design diagrams for wave overtopping rate at a vertical wall, to display wall freeboard required for rates of 1 cfs/ft and 0.01 cfs/ft with various incident wave heights. Reference 26 also provides a convenient summary on the effect of appreciable fronting roughness in storm waves: the required freeboard of a smooth vertical wall for a given overtopping rate is about 1.5 times that needed when a sizable mound having concrete block armor is installed against the wall. With this information, a specific vertical wall can be categorized as having only modest overtopping ($\overline{Q}$ < 0.01 cfs/ft),

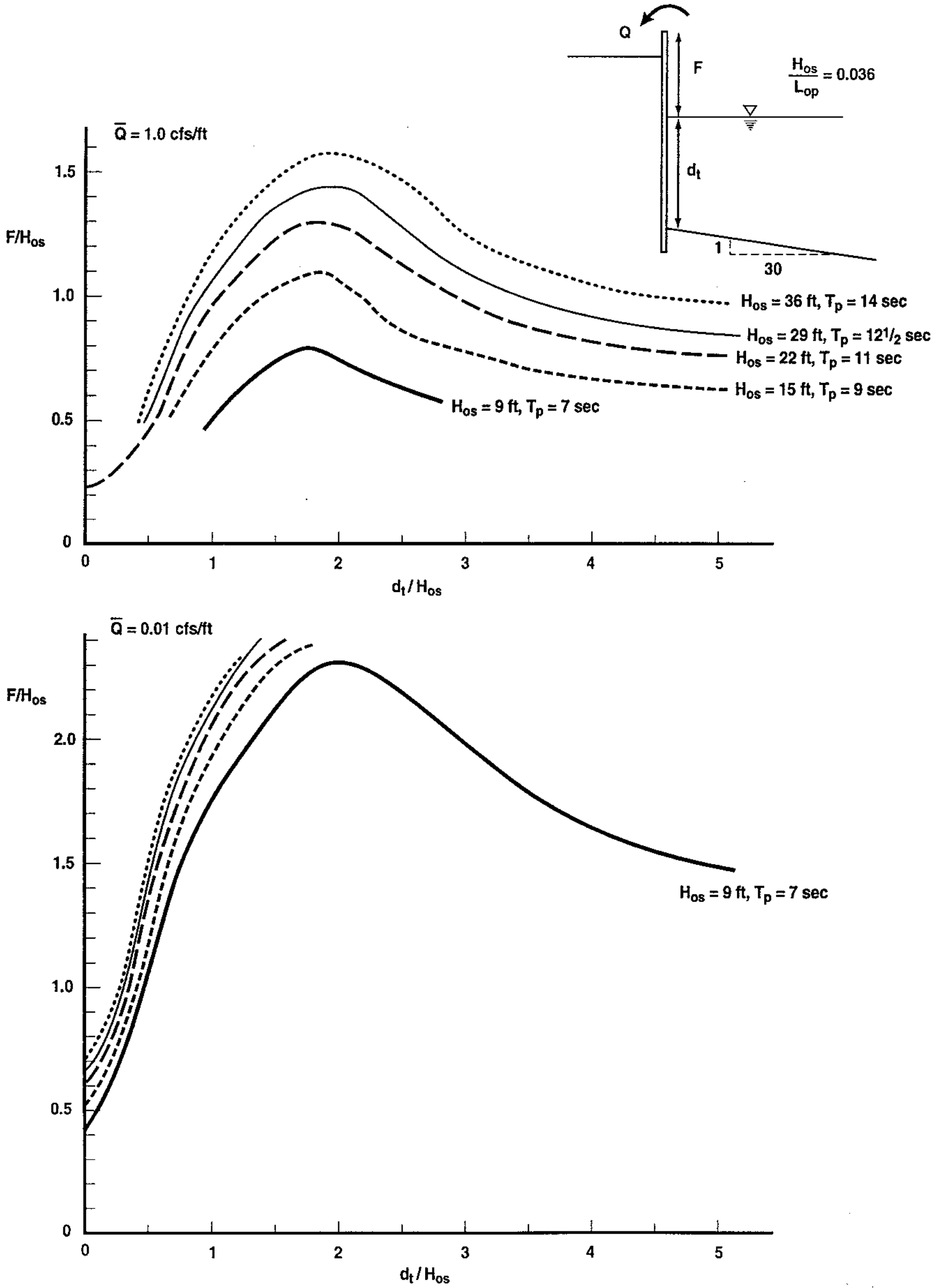

Figure 20. Required Freeboard of Vertical Wall to Limit Mean Overtopping Rate to Certain Values, Based on Design Curves of Reference 26.

intermediate overtopping, or severe overtopping ($\overline{Q}$ > 1 cfs/ft) expected for the base flood. Likely runoff or ponding behind the wall then needs to be identified, and severe overtopping requires delineation of the landward area having wave impacts and velocity hazard. Chapter 7 outlines some common zonations of flood hazards near shore barriers in describing the integration of computed wave effects.

Considering Figure 20 along with common wall and wave heights, wave overtopping dangerous to structural stability appears the usual case in the base flood. An assessment of failure during the base flood for typical walls would be fully consistent with one recommendation of Reference 17, namely that "FEMA not consider anchored bulkheads for flood-protection credit because of extensive failures..."

Interpretation of estimated overtopping rate in terms of flood hazards is complicated by the projected duration of wave effects, by the increased discharge possible under storm winds, by the varying inland extent of water impacts, and by the specific topography/ drainage landward of the barrier. However, guidance in Table 7 is provided as potentially applicable to typical coastal situations.

For each coastal structure experiencing sizable wave runup in the base flood (say, $\overline{R}$ > 2 ft), a brief report to the Project Officer should outline overtopping assessments, and document conclusions consistent with historical evidence for the site.

Table 7. Suggestions for Interpretation of Mean Wave Overtopping Rates

$\overline{Q}$ Order of Magnitude	Flood Hazard Zone Behind Barrier
<0.0001 cfs/ft	Zone X
0.0001-0.01 cfs/ft	Zone AO (1 ft depth)
0.01-0.1 cfs/ft	Zone AO (2 ft depth)
0.1-1.0 cfs/ft	Zone AO (3 ft depth)
>1.0 cfs/ft*	30-ft width+ of Zone VE (elevation 3 ft above barrier crest), landward Zone AO (3 ft depth)

*With estimated $\overline{Q}$ much greater than 1 cfs/ft, removal of barrier from transect representation may be appropriate.

+Appropriate inland extent of velocity hazards should take into account structure width, incident wave period or wavelength, and other factors.